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HYDRAULICS AND HYDROLOGY

A.1. GENERAL

This appendix presents detailed descriptions of the climatology and hydrologic regimen of the area and detailed descriptions of hydraulic analysis methods and procedures used in the design of the protection features of the plan. These descriptions include essential data, assumptions, and criteria used in the study that provides the basis for determining surges, routings, wind tides, wave runup and overtopping, and stage frequencies. Designs for protective structures at elevation +6.0 feet, +7.0 feet, and +8.0 feet National Geodetic Vertical Datum (N.G.V.D.) were developed. Parameters for various frequency storms were derived from the Standard Project Hurricane (SPH) using methodology furnished by the National Weather Service and differ from the SPH only in central pressure index and windspeed.

The study area is located in Jefferson Parish, west of the Mississippi River within the area known as the Barataria Basin. The Bayou Lafourche ridge bound the Barataria Basin to the west, the Mississippi River to the north and east and the Gulf of Mexico to the south. Lakes Salvador and Cataouatche are estuary areas to the west, which connect to the Gulf of Mexico through Barataria Bay. Tidal waters are carried into the study area through these lakes and Bayou Barataria into the Harvey, Algiers and Hero Canals. Freshwater is introduced into the study area from the Mississippi River via the Harvey and Algiers Locks, direct rainfall and pumped discharges from leveed areas.

A.2. TERRAIN

The Fisher School Basin, located in southeastern Louisiana, is of mostly low relief and characteristic of an alluvial plain. Situated on the eastern bank of Bayou Barataria near New Orleans, land elevations slope gently from an average elevation of about 4 feet NGVD along the natural banks of Bayou Barataria to approximately one foot below sea level in portions of the study area. Natural ground elevations in the unprotected marsh areas in the eastern part of the study area average 0.5 to 1.0 feet NGVD. Although leveed marshland will subside when pumped, unleveed areas are subject to natural subsidence and in the future will become increasingly vulnerable to flooding from the combined effects of this subsidence and eustatic/global sea level rise. Within the study area 0.5 feet of subsidence was assumed throughout most of the area during a 100-year period; along the eastern part of the study area from 0.6 to 1.2 feet of subsidence is expected. Sea level rise is assumed to be 0.5 feet in 100 years.

All of the area is protected from Mississippi River overflows by the mainline levee system. Flooding originating in the Gulf of Mexico and Lakes Salvador and Cataouatche can travel across the marsh and through the many natural and man-made channels to inundate the Fisher School Basin from the south. To protect the area from this tidal and storm surge flooding, local interests have constructed a partial levee. The levee begins at the south-eastern end of the basin at Louisiana Highway 45 (LA 45) and proceeds along several man-made canals along the eastern end of the alignment and gradually declines in elevation to the existing ground approximately 2500 feet from LA 45 in the north, near Fleming Curve. The naturally high-ridge along Bayou Barataria varies in elevation from +4.0 ft to +1.0 ft NGVD and provides marginal protection from high tidal stages along the northern and western sections of the study area. The existing levee and natural ridges do not form a closed flood protection system for the Fisher Basin.

Rainfall amounts used to estimate interior flooding elevations and design drainage structures were taken from the National Weather Service Technical Paper (TP) 40, which gives rainfall totals for various durations and frequencies across the United States. In the design studies, rainfall amounts for the design rainfall included lesser duration rainfalls. For instance, imbedded in the 100-year, 24-hour rainfall distribution are the 100-year, 1-hour, 2-hour, 3-hour, 6-hour, and 12-hour rainfall amounts, as given in TP 40. This methodology is used to determine each area's sensitivity to the various durations of more intense rainfalls. Similar distributions of duration can be applied to any frequency of rainfall, as depicted by TP 40.

A.3. CLIMATOLOGY

a. Climate. The Fisher School Basin has a subtropical marine climate. Located in subtropical latitude, its climate is influenced by the many water surfaces of the lakes, streams, and the Gulf of Mexico. Throughout the year, these water bodies modify the relative humidity and temperature conditions decreasing the range between the extremes. When southern winds prevail, these effects are increased, imparting the characteristics of a marine climate.

The area has mild winters and hot, humid summers. During the summer, prevailing southerly winds produce conditions favorable for afternoon thundershowers. In the colder seasons, the area is subjected to frontal movements that produce squalls and sudden temperature drops.

b. Temperature. Records of temperature are available from "Climatological Data" for Louisiana, published by the National Climatic Center. The study area can be described by using temperature data observed at LSU Citrus Research Station in Plaquemine Parish. The annual normal temperature of this station based on the period 1961-1990 is 60.1 degrees Fahrenheit (^{o}F) with monthly mean temperature normals varying from 42.5 ^{o} F in January to 73.7^{o} F in July. Temperature normals are shown in Table A-1 and the extremes of this station since 1984 are shown in Table A-2.

TABLE A-1

MEAN MONTHLY and ANNUAL TEMPERATURE (°F)

30 Year Normals (1961-1990)

	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANN
LSU CITRUS	42.5	45.1	51.9	60.2	67.0	72.5	73.7	73.6	71.6	62.4	54.1	46.4	60.1

Source: National Climatic Center

TABLE A-2

TEMPERATURE EXTREMES (°F) 1984-1992

STATION	MAXIMUM	DATE	MINIMUM	DATE
LSU CITRUS	97	*	12	23 DEC 89

* Occurring on several days.

Source: National Climatic Center

c. Precipitation. The annual normal precipitation for the study area based on National Climatic Center records at LSU Citrus Research Station over the period 1961-1990 is 62.85 inches. Table A-3 lists the monthly and annual normals. The maximum monthly rainfall and greatest day of this station since 1984 is shown in Table A-4. There have been some months that recorded no precipitation. The heaviest rainfall usually occurs during the summer with July being the wettest month with an average monthly normal of 6.82 inches. October is the driest month, averaging 3.40 inches.

TABLE A-3
MONTHLY PRECIPITATION (inches)
30 Year Normals (1961-1990)

	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANN
LSU CITRUS	5.05	5.83	4.99	4.06	5.08	5.59	6.82	6.67	5.89	3.40	4.26	5.21	62.85

Source: National Climatic Center

TABLE A-4
MAXIMUM PRECIPITATION TOTALS
(inches)(1984-1992)

Station	Maximum Monthly	Date	Greatest 1 Day	Date
LSU CITRUS	20.00	APR 91	8.73	2 AUG 84

Source: National Climatic Center

d. Wind. Wind data taken at New Orleans is used to describe the study area. The average wind velocity is 8.0 miles per hour (mph) over the period 1973-1992. Southeast winds predominate in the spring and summer. The prevailing winds of the fall and winter are from the northeast. Winter storms in the area have produced wind speeds of up to 47 mph. The summer is often disturbed by tropical storms and hurricanes that produce the highest winds in the area. The maximum wind speeds observed (highest one-minute speed) since 1963 was 69 mph at New Orleans and the result of Hurricane Betsy in September 1965.

e. Stream Gaging Data. Records of stage data are available at two stations within the study area. Discharge measurements are not available due to tidal influence. Stream gaging data such as period of record, maximum and minimum extremes are presented below in Table A-5.

TABLE A-5

STREAM GAGING DATA

MAP STATION NO.	PERIOD OF RECORD	MAXIMUM STAGE		MINIMUM STAGE	
		FT NGVD	DATE	FT NGVD	DATE
1 BAYOU BARATARIA @BARATARIA	1950-92	4.25*	29 OCT 85	-0.58*	10 SEP 65
2 BAYOU BARATARIA @ LAFITTE	1963-92	5.05*	29 OCT 85	-0.95a	23 DEC 89

* Caused by Hurricane/Storm

a From incomplete record

Source: U.S. Army Engineers District, New Orleans

f. Floods and Storms of Record. Most of the flooding in the study area is from high tides caused by hurricanes and tropical storms tracking in the Gulf of Mexico. Some of the major storms that have passed through or near the study area are shown below in Table A-6.

TABLE A-6
EXPERIENCED HURRICANES

STORM	DATE	MAXIMUM CENTRAL PRESSURE (Inches Mercury)	MAXIMUM FORWARD SPEED(Knots)	MAXIMUM RECORDED WINDSPEED (M.P.H.)
1915	22 Sep-2 Oct 1915	27.87	10	94
1947	4-21 Sep 1947	28.57	16	98
FLOSSY	21-30 Sep 1956	28.76	20	90
HILDA	28 Sep – 5 Oct 1964	28.4	7	98
BETSY	27 Aug-10 Sep 1965	28.0	20	105
CARMEN	29 Aug–10 Sep 1974	27.84	9	86
BABE	3-8 sep 1977	29.85	-	75
BOB	9-16 Jul 1979	29.58	15	75
DANNY	12-20 Aug 1985	29.61	13	85
JUAN	26-31 Oct 1985	29.13	13*	74
ANDREW	16-28 Aug 1992	27.66	15	150

* Maximum reported forward speed. Several times during its traversal, the storm stalled while changing direction.

Hurricane Flossy brought torrential rains and tidal flooding to the study area. Golden Meadow, which is below the study area, received 16.7 inches of rain in a 24-hour period. Hurricane Hilda raised water levels at Barataria and Lafitte to 3.6 and 4.0 feet, NGVD, respectively. Hurricanes Betsy and Carmen also caused flooding to some parts of the study area. Hurricane Juan broke high water records at both gages in the study area (see Table A-5). Flooding was from tidal inundation and high stages caused by Juan's prolonged stay. Total storm precipitation for Juan ranged from 8-12 inches over the area. Hurricane Andrew, which was the last storm to hit the Louisiana coast raised water levels at Barataria and Lafitte to 3.5 and 4.2 feet NGVD, respectively.

Other flooding in the area is from a combination of high gulf tides and runoff from heavy rainfall. An example of this flooding occurred in the spring (Apr-May) of 1991 when Bayou Barataria at Barataria recorded a peak stage of 3.4 feet NGVD and Bayou Barataria at Lafitte recorded a peak stage of 3.32 feet NGVD, both on 29 April 1991.

g. Tides. Tides in the study area can be diurnal or semi-diurnal depending on astronomical conditions. The tidal range at Barataria is 0.25 feet, NGVD, with the mean high water being approximately 1.47 feet, NGVD, and the mean low water approximately 1.22 feet, NGVD. The highest observed stage at Barataria was 4.25 feet, NGVD (29 Oct 85), and the lowest observed stage was -0.58 feet, NGVD (9 Sep 65). At Lafitte, the tidal range is 0.35 feet, NGVD, with the mean high water measuring approximately 1.49 feet, NGVD, and the mean low water approximately 1.14 feet, NGVD. The highest observed stage was 5.05 feet, NGVD (29 Oct 85), and the lowest observed stage was -0.68 feet, NGVD (25 Dec 85).

A.4. DESIGN STORM.

Protective structures at elevation +6.0 ft, +7.0 ft, and +8.0 ft NGVD were analyzed by running storm events that range in frequency from 1 year to 500 years. The SPH (Standard Project Hurricane) represents the most severe combination of hurricane parameters that is reasonably characteristic of the area, excluding extremely rare combinations. The hurricane would approach each individual site at such a rate of movement as to produce the maximum hurricane surge at each location of interest. The SPH has a central pressure index of 27.4 inches of mercury, a maximum 5 minute average wind velocity offshore (in the Gulf of Mexico) of 100 knots 30 feet above the surface at a radius of 30 nautical miles, and a forward speed of 11 knots along a path critical to each location of interest. The 100- and 10-year frequency storms were derived from the SPH parameters using experienced stage frequencies and data provided by the National Weather Service. Hurricane parameters for other frequency storms differ from the SPH only in central pressure index and windspeed.

A.5. DESCRIPTION AND VERIFICATION OF PROCEDURES.

a. Hurricane Memorandums. The Hydrometeorological Section (HMS) of the National Weather Service has cooperated in the development of hurricane criteria for experienced and potential hurricanes in the study area. The HMS memorandums provided isovel patterns, hurricane paths, pressure profiles, rainfall estimates, frequency data, and various other parameters required for the hydraulic computations. A reevaluation of historic meteorlogic and hydrologic data was the basis for memorandums relative to experienced hurricanes. Those relative to potential hurricanes were developed through the use of generalized estimates of hurricane parameters based on recent research and concepts of hurricane theory. Memorandums applicable to the study area are listed in the attached bibliography.

b. Surges. Maximum hurricane surge heights along the gulf shores were determined from computations made for ranges extending from the shores out to the continental shelf by use of a general wind tide formula based on the steady state conception of water superelevation (1)(2)(3)*. The average windspeed and average depth in each range were determined from isovel and hydrographic charts for each computation. The National Weather

Service furnished the storm isovel patterns. In order to reach agreement between the computed maximum surge heights and the observed high water marks, it was necessary to introduce a surge adjustment factor or calibration coefficient into the general equation, which in its modified form, was as follows:

$$S = 1.165 \times 10^{-3} \frac{V^2 FNZ}{D} \cos 0$$

Where, S = wind setup in feet

V = windspeed in m.p.h.

F = fetch length in statute miles

D = average depth of fetch in feet

N = planform factor, assumed equal to unity

Z = surge adjustment factor

0 = angle between direction of wind and the fetch

* Numbers in parenthesis indicate reference in bibliography

Hurricane surges at the shore were determined by summation of incremental wind setups along a range above the water surface elevation at the gulf end of the range. A combination of the setup due to atmospheric pressure anomaly and the predicted normal tide was used to determine the initial elevation at the gulf end of the range. Due to the variation in pressure setup between the shoreward end and gulfward end of the range, an adjustment was made at the former to compensate for the difference. This procedure for determining surge heights at the coastline was developed for the Mississippi Gulf Coast, where reliable data was available at several locations for more than one severe hurricane, and is used for the entire coastal Louisiana region. Due to dissimilar shoreline configurations, different factors were required at different locations, but identical factors were used at each location for every hurricane. The value of the factor is apparently a function of the distance from the shoreline to deep water and varies inversely with this distance. Comparative computed surge heights and observed high water marks for the 1915 and 1947 hurricanes at the locations used to verify the respective procedures are shown in Table A-7. All elevations in this appendix are in feet and are referred to National Geodetic Vertical Datum of 1929 (NGVD).

In those areas where a coastal bay separated from the gulf by an offshore barrier island such as Grand Isle characterizes the coastline or by a shoal, it is necessary to inject an additional step in the normal procedure to verify experienced hurricane tides. The incremental step computation was completed to the gulf shore of the island and the water surface elevation transposed to the inland bay side of the island from whence the incremental computations were continued using a new surge adjustment factor that was considered representative of the shallower depths within the bay. This procedure resulted in a satisfactory verification of hurricane tides along other portions of the Louisiana coast.

The incremental step computation was used to check elevations experienced during the hurricane of 22 September - 2 October 1915 and Hurricane Flossy, 21-30 September 1956. Verification of surge heights and surge adjustment factors for these hurricanes are shown in Table A-8. Surge adjustment factors of 0.80 in open water and 0.48 in Barataria Bay were used for the Manila Village area.

TABLE A-7

HURRICANE SURGE HEIGHTS

Location	Surge Adjustment Factor (Z)	1915		1947	
		Observed	Computed	Observed	Computed
		(feet NGVD)		(feet NGVD)	
Long Point, La.	.21	9.8	9.6	10.0	10.1
Bay St. Louis, Ms.	.46	11.8	11.8	15.2	15.1
Gulfport, Ms.	.60	10.2*	9.9	14.1	14.3
Biloxi, Ms.	.65	10.1*	9.8	12.1*	12.6

* Average of several high water marks.

TABLE A-8

VERIFICATION OF HURRICANE SURGE HEIGHTS

Location	Surge Adjustment Factor (Z)	Sep 1915		Sep 1947 (Flossy)	
		Observed	Computed	Observed	Computed
		(feet NGVD)		(feet NGVD)	
Grand Isle					
Flooding from front	0.80(a)	9.0	8.8	3.9	4.1
Flooding from rear	0.80(a)	-	-	8.0	7.8
Manila Village	0.48(b)	8.0	8.5	-	5.1

(a) In Gulf of Mexico

(b) In Barataria Bay

c. Routing. Since the major hurricane damage in the study area would result from storm induced effects on Lake Salvador, it was necessary to establish a method to determine the stage in the lake at any time during the hurricane occurrence. This procedure involves the construction of a stage hydrograph for Barataria Bay by calculating the hourly flows and rainfall simultaneously through Lake Salvador's natural inlet channels (assumed in this case to be one large channel).

Prerequisite to any routing is the choice of an actual or hypothetical hurricane of known or designated characteristics.

It is then possible to develop surge heights for any point in Barataria Bay for the selected hurricane. For routing purposes, Manila Village, which is about 20 miles southeast of Lake Salvador was selected as the critical point for a hydrograph. It would reflect stages at the mouth of the schematized inlet channel. Such a hydrograph of hourly stages was constructed by computing the incremental setup for each hour and using the maximum surge elevation as the peak of the hydrograph for the critical period. Storm surge hydrographs at Manila Village for other frequencies were determined by identical procedures.

A stage area curve was made for the schematized conveyance channel between Manila Village and the entrance to the Lake Salvador Basin, which consists of Lake Salvador, Lake Cataouatche, and the adjacent marsh area. Since the width of the channel is very large, the depth of water was used as the hydraulic radius.

The cumulative amount of rainfall coincident with the storm significantly affects the lake elevation and, therefore, the routing procedure. The amount of this rainfall was calculated by the methods described in U.S. Weather Service memorandums (4)(5), using a moderate rainfall that would be coincident with a tropical storm. For routing purposes, a moderate rainfall of 8.50 inches in 24 hours was considered as additional inflow into the Lake Salvador Basin. The effect of cumulative rainfall is to raise the average lake level.

With the above mentioned items resolved, the routing procedure was reduced to the successive approximation type problem in which the variable factors were manipulated until a correlation between flows from the gulf through the inlet channel and the rise in the mean elevation of the Lake Salvador Basin was obtained for the incremental time intervals. The use of this method was illustrated by Bretschneider and Collins (6). For verification of the method, the surge caused by Hurricane Betsy, September 1965 was routed by this procedure. The routed stage for Bayou Barataria at Lafitte (assumed to be the representative stage of the Lake Salvador Basin) was found to be in reasonable agreement with the observed stage for the hurricane. The observed and computed peak stages for Hurricane Betsy are 3.35 and 3.05 feet, respectively. If the average stage between the Lafitte and Barataria, Louisiana were used as the representative stage, the computed and observed stages would be in very close agreement.

d. Wind Tides. When strong hurricane winds blow over enclosed bodies of shallow water, they tend to drive large quantities of water ahead of them. Therefore, wind tide levels (WTL's) in Lakes Salvador and Cataouatche, respectively, are needed to determine stage damage curves and to design protective levee heights.

Lakes Salvador and Cataouatche are located in a marsh west of the study area and are so situated that the volume of incoming flow from the gulf cannot be measured because the water flows over broad areas of ungaged marshland. Therefore, the extensive marshlands that surround both lakes results in an almost unlimited storage area when lake waters overflow their banks. Hourly lake elevations for the various frequencies used in computing wind tide levels for Lakes Salvador and Cataouatche were obtained from the routed hydrographs that reflect the average lake level.

To compute wind tide, the lake is divided into three zones roughly parallel to wind directions. A nodal line is designated perpendicular to the zones and setup is calculated for the leeward segment and setdown for the windward segment. The average windspeed and average depth in each segment were determined from isovel and hydrographic charts for each computation. The storm isovel patterns were furnished by the U.S. Weather Service (ESSA)(7). The computation of setup or setdown along each segment was based on the segmental integration method (3) and was calculated by the use of the step method formulas (8) that were modified as follows:

$$Setup = d_t\left(\sqrt{\frac{0.00266u^2FN}{d_t^2}+1}\ -1\right)$$

$$Setdown = d_t\left(1-\sqrt{1-\frac{0.00266\,u^2FN}{d_t^2}}\right)$$

Where: setup or setdown in feet is measured above or below mean water level (mwl) of the surge in the lake.

d = average depth of fetch in feet below m.w.l.
u = windspeed in m.p.h. over fetch.
F = fetch length in miles, node to shoreline.
N = planform factor, equal generally to unity.

Graphs were constructed from the above formulas to determine setup and setdown quickly about the nodal elevation for storms of varied frequencies. Volumes of water along the zones, represented by the setup and setdown with respect to a nodal elevation, were determined and the water surface profiles adjusted until setup and setdown volumes for the lake balanced within 5 percent. Then setup elevations were added to the still water level to yield the WTL. The time dependent SPH and Design Hurricane wind tide hydrographs were computed for the eastern and northern shore of Lakes Salvador and Cataouatche.

Observed wind tide elevations at the shorelines of Lakes Salvador and Cataouatche are not available. Therefore, the method of wind tide level computation could not be verified by comparing observed and computed data. However, the above-described method has been used successfully for the south shore of Lake Pontchartrain at New Orleans, Louisiana. Observed data were available for this lake and the method verified.

In order to obtain wind tide levels along Louisiana Highway 45, it was necessary to use the relationship between the maximum wind tide level and the distance inland from the shoreline.

Marshlands that fringe the shoreline in certain locations are inundated for considerable distances inland by hurricane wind tides that approach the shores. The limit of overland surge penetration depends upon the height of the wind tides and the duration of high stages at the lakeshore. The study of available observed high water marks at the coastline and inland indicates a fairly consistent simple relationship between the maximum surge height and the distance inland from the coast. This relationship exists independently of the speed of hurricane translation, wind speeds, or directions. The data indicates that the weighted mean decrease in surge heights inland is at the rate of 1.0 foot per 2.75 miles. This relationship remains true even in the western portion of Louisiana where relatively high chenieres, or wooded ridges, parallel the coast. Efforts to establish time lags between peak wind tide heights at the shoreline and at inland locations were unsuccessful because of inadequate basic data.

For the purpose of surge routing procedures, the shoreline is defined as the locus of points where the maximum WTL's would be observed along fetches normal to the general shore. This synthetic shoreline is assumed to be along the southern portion of the Lake Cataouatche levee and near the extreme western side of the Bayou Des Familles ridge. In order to determine the maximum water surface elevations at inland locations, it was necessary to compute maximum WTL's at the designated points mentioned above. These computed wind tide levels were then adjusted by application of the average slope of maximum surge height inland (1 foot/2.75 miles) to the location of interest. Hurricane stages were not available for positive verification of the procedure within the area. However, the procedure has given satisfactory results in this area and has verified the observed data in other areas of study with similar topography and bathymetry.

A.6. LEVEES.

The mainline Mississippi River and Tributaries levee system protect the study area from river overflow. A partial levee along the eastern end of the study area provides some protection from tidal stages. The levee was constructed by local interests as expanding development-demanded protection following severe storm events. The levee varies in elevation from +2.5 to +4.0 feet NGVD. Along the western and northern sections of the study area, the Bayou Barataria bankline varies in elevation from approximately +1.0 feet to +4.0 feet NGVD.

The existing levee ties into LA 45 at the southeastern end of the Fisher School Basin, however it gradually slopes to the natural ground elevation of +2.5 ft NGVD in the northeastern end of the study area. The levee does not tie into the streambank, therefore the Fisher Basin is not protected by a closed system. The integrity of the local levees is questionable in view of failures that occurred to similar levees west of the Harvey Canal during Hurricane Juan.   Variations in elevation cause frequent overtopping of the existing protection.

For with-project conditions, a closed levee system at elevation 6.0 ft, 7.0 ft, and 8.0 ft NGVD was considered for this area. Waves larger than the significant wave may overtop the protective structures, but, due to the limited number of waves larger than the significant wave, such overtopping will not endanger the security of the structure. Where levees or floodwalls are sheltered from storm-generated wave runup, wave runup from small locally generated waves, which cannot be predicted from our standard methodology, can overtop the levee. For this study 1-foot waves with small periods, 2.7 seconds, were used to compute runup from these small unpredictable waves. Methods used for computing wave runup are explained in the Shore Protection Manual, published by the Coastal Engineering Research Center in 1984. Wave runup of 2 feet was determined for the sheltered reaches of levee. Design elevations for the protective structures in each reach for the alternatives studied are shown in Table A-9.

TABLE A-9

DESIGN ELEVATION OF PROTECTIVE STRUCTURES

Location	SWL ft	WAVE RUNUP	10-Year
Bayou Barataria Floodwall	3.8	2.0	6.0
Eastside Levee	3.8	2.0	6.0

A.7. STAGES AND DURATIONS.

Extreme astronomical high tides accompanied by heavy rainfall and/or storms can cause flooding in the study area. Extended duration weak hurricanes, such as Hurricane Juan, can produce a storm surge of sufficient height to overtop existing protective embankments and flood the heavily populated developed areas.

In 1973, floodwaters resulting from excessive rainfall and abnormally high tides in Lakes Cataouatche and Salvador and Bayou Barataria prevented adequate drainage and caused damage to residential areas.

Drainage problems are exacerbated when rainfall is accompanied by high tides. During May 1978 and April 1980, short duration, large accumulation rainfalls occurred in this area. During the rainstorm of 3 May 1978, the stage was 2.3 feet NGVD at Barataria on Bayou Barataria and 2.7 feet NGVD at the Harvey Lock on the Intracoastal Waterway because of strong onshore winds that accompanied the rainstorm. At the city of Algiers, 9.8 inches of rainfall were measured. On 13 April 1980, the rainfall measured at Algiers was 9.7 inches and the accompanying stage at Barataria was 3.8 feet NGVD. At the

Harvey Lock, the maximum stage was 3.2 feet NGVD. Pump stations that discharge into the marsh were forced to operate against higher than optimum outside stages during these events, reducing the capacity of these stations.

Continuous records of stages are available at several locations in and near the study area. On the westbank of Jefferson Parish, several continuous gages were operated: Bayou Barataria at Barataria from 1950 to 1992, Bayou Barataria at Lafitte from 1963 to 1992, and Bayou Rigaud at Grand Isle from August 1947 to the present. A recording gage for hurricane stages is located on Grand Isle at the mayor's office. A wire-weight type gage, located in the Intracoastal Waterway at the Harvey Lock, is read daily, usually at 8 a.m. Records for this gage are available from January 1925. Another wire-weight gage is located, along with a continuous gage, in the Intracoastal Waterway at Algiers Lock; it is read daily at 8 a.m. Records are available at this location from 1956. In the Mississippi River, the continuous gage located nearest Jefferson Parish is the Carrolton Gage located in Orleans Parish at River Mile 102.8; it has been in operation since January 1872. All of these gage records are published annually in "Stages and Discharges of the Mississippi River and Tributaries." In addition, gage information and stillwater elevations for hurricanes of relatively recent history affecting the area are available in various other publications of the U. S. Army Corps of Engineers and other agencies.

Intense hurricanes such as Betsy have caused high stages along the coastal area of Louisiana (10.5 Feet NGVD at Grand Isle) and moderately high stages inland (3.2 feet NGVD at the Harvey Lock). High stages resulting from several hurricanes are summarized in the section on "Hurricanes and Tropical Storms" in this report. Detailed data is presented in a Corps publication entitled, "History of Hurricane Occurrences along Coastal Louisiana." Examination of gage records at the inland gaging stations reveals that Hurricane Juan caused the highest stage of record on 29 October 1985, along Bayou Barataria at both Barataria (4.25 feet NGVD) and Lafitte (5.05 feet NGVD) and at the Algiers (4.45 feet NGVD) and Harvey (4.74 feet NGVD) Locks.

The normal tide in the study area is diurnal. However wind effects can mask the daily ebb and flow variations and during periods of sustained southerly winds, tides rise in direct response to the duration and intensity of the wind stress. Hurricane Juan demonstrated this in 1985. Although a relatively weak storm in terms of maximum sustained windspeed, Hurricane Juan caused higher stages in much of the study area than the more intense Hurricane Betsy. This is directly attributable to the hurricane's erratic, almost stationary, path across southern Louisiana. Gale force winds over a period of 5 days caused tides 3 to 6 feet above normal across the entire coastal area of southern Louisiana.

A.8. FREQUENCIES.

To determine the design stages for the study area, frequency estimates were developed for experienced hurricane stages and analysis of theoretical hurricane stages. Using stages measured at the gaging stations in the study area, an experienced stage frequency curve was drawn for each station for the combined effects of hurricane induced storm surge and high stages caused by other events, using procedures outlined in EC 1110-2-249, Hydrologic Frequency Analysis.

To develop characteristics for the design hurricanes, information on hurricanes published by the National Weather Service was used. The National Weather Services made a generalized study of hurricane frequencies and parameters and presented the res0ults in NOAA Technical Report NWS23, "Meteorological Criteria for Standard Project Hurricane and Probable Maximum Hurricane Windfields, Gulf and East Coasts of the United States, September 1979"(9). In a 400 mile zone along the central gulf coast from Cameron, Louisiana, to Pensacola, Florida (Zone B), frequencies for hurricane central pressure indexes (CPI) presented in the report reflect the probability of hurricane recurrence in the mid-gulf coastal area. Hurricane characteristics with critical tracks and CPI's representative of the SPH were developed in cooperation with the National Weather Service. The CPI used was 27.45 inches for this hurricane. The SPH described in NHRP Report No. 33, and NWS Report 23 was the basis of development of the Design Hurricane used in the study.

The Standard Project Hurricane is a large storm of moderate forward speed and high wind speed. Relatively weak storms, such as Hurricane Juan, have weak steering currents and historically are the storms that will stall. An intense hurricane, such as Betsy or Camille, has strong steering currents and moves at a moderate to fast forward speed, making landfall with few changes in course. For these reasons, the SPH was assumed to travel at a moderate forward speed without stalling.

Hurricane Wind Tide Levels (WTL'S) were computed for the theoretical hurricanes in accordance with prescribed procedures for determining setup and setdown in an enclosed lake. Isovels were rotated and the path transposed within allowable limits as necessary to produce maximum surge elevations at the proposed levee.

A synthetic stage frequency curve was developed by correlating stages and frequencies for corresponding CPI's, using a procedure developed for the Lake Pontchartrain study area. Experienced stage frequency curve developed at the gaging station in Bayou Barataria was used to adjust synthetic stages in these canals. Stages for study area that would accompany the SPH, 100-year and 10-year storms are shown in Table A-10.

TABLE A-10
COMPARATIVE SURGE HEIGHTS

	Stages in feet NGVD		
Location	SPH	100-year	10-year
Bayou Barataria	9.0	7.0	3.8

A one-dimensional model was used to develop the frequency curves for this project. The project has not been redesigned using a two-dimensional model. However, the two-dimensional numerical model, WIFM, was used to compute water surface elevations in the Barataria Basin. The WIFM model, developed by the Waterways Experiment Station (WES), was calibrated by them for the Louisiana coastal area and used extensively for computing hurricane surges in the coastal region and areas adjacent to Lake Pontchartrain. The results from the WIFM model, using the design SPH as the forcing function, verify the mean stages computed with the calibrated one-dimensional model for Lakes Cataouatche and Salvador as well as open coast surge heights at Grand Isle and Venice. Therefore, no further studies using this two-dimensional model were undertaken for this area.

The probability value used for a given CPI represents frequency of occurrence from any direction in a 400-mile zone along the central gulf coast. In order to establish frequencies for the locality under study, it was assumed that hurricanes critical to the locality would pass through a 50-mile subzone along the coast. Thus, the number of occurrences in a 50-mile subzone would be 12.5 percent of the number of occurrences in a 400-mile zone, provided that all hurricanes traveled in a direction normal to the coast. A hurricane whose track is perpendicular to the coast ordinarily will cause extremely high tides and inundation for a distance of about 50 miles along the coast. However, the usual hurricane track is oblique to the shoreline. The average projection along the coast of this 50 mile swath for the azimuth of 48 Zone B hurricanes is 80 miles. Since this is 1.6 times the width of the normal 50-mile strip affected by a hurricane, the probability of occurrence of any hurricane in the 50-mile subzone would be 1.6 times the 12.5 percent of the probabilities for the entire mid-gulf Zone B. Therefore, 20 percent of the frequencies of hurricanes for Zone B, mid-gulf, was used to represent the frequencies of hurricanes in the critical 50-mile subzone for each study locality.

Since tracks having major components from the southeast create the most critical stages in the Grand Isle area, maximum hurricane surge heights were computed for synthetic hurricanes approaching the area on a track from that direction. Four-fifths of all tracks that approached the Grand Isle area were from the southeast. Therefore, a stage frequency curve was derived using 4/5 of the 50-mile subzone probability for all tracks. Frequencies for observed hurricane stages were then computed on the same basis as the CPI frequencies (10), and a curve plotted. The synthetic frequency curve was then adjusted and plotted to the Grand Isle observed data. A frequency curve for Manila Village was then obtained by adding the additional wind tide setup across Barataria Bay to the appropriate stage frequency value on the adjusted Grand Isle curve.

There is a direct relationship between the stage frequency at Manila Village and the average stage frequency in Lakes Salvador. However, the critical stage frequency at the shoreline is considerably diminished because the hurricane track required to cause critical stages at the eastern shore of Lake Salvador is unique. Only 6.4 percent of all hurricane tracks observed have followed a track similar to the unique hypothetical track used in this study. Stage frequencies were also developed based on the remaining 93.6 percent-observed hurricane tracks.

The azimuths of tracks observed in the vicinity of the study area were divided into quadrants corresponding to the four cardinal points. Since 1900, 73 storms have affected the Louisiana coast; 46 had tracks from the south, 18 from the east, 8 from the west, and 1 from the north. Hurricanes with tracks having major components from the south and east generate WTL's that are near critical relative to the study area, while those tracks from the west generate WTL's most critical to the study area. The average azimuth of tracks from the south is 180 degrees. Tracks from the east had an average azimuth of 117 degrees. These azimuths, along with the critical track from the west, were used in computing WTL's for Lake Salvador. Of all experienced tracks since 1900 affecting the Louisiana Coast, approximately 63 percent have come from a southerly direction, 24.6 percent from the east, and 11 percent have come from the west. The probabilities of equal stages for the three groups of tracks were then added arithmetically to develop a curve representing a synthetic probability of recurrence of maximum wind tide levels for hurricanes from all directions. Table A-11 illustrates the synthetic frequency computation for WTL's at the east shore of Lake Salvador. Using these procedures, stage frequency relationships were established under existing conditions for flooding by surges from Lakes Salvador for the area along Highway 45 between Crown Point and Lafitte, Louisiana. See Plate A-1 for stage-frequency curves for Bayou Barataria at Lafitte, without project conditions for Fisher School-Fleming Curve Basins, and with project conditions for Fisher School-Fleming Curve Basins.

A.9. FUTURE CONDITIONS.

Historical evidence of sea level rise and subsidence indicates the need for a projection of storm surge stages and their effect on this project's effectiveness. Sea level rise of 0.5 feet per century along the Gulf Coast is recommended by the latest Corps' guidance. COE geologists from radio carbon dating of buried marsh deposits developed estimates of subsidence in coastal Louisiana. This data was compiled on quadrangle maps for coastal Louisiana. Using the projected sea level rise of 0.2 feet in the next 50 years and the appropriate subsidence rate in the coastal zones bordering the project area, the WIFM model was employed to compute the hurricane surge heights which could be expected in the year 2040. Stages for pertinent locations in the area that would accompany the SPH, 100-year and 10-year hurricanes are shown in Table A-12.

Table A-11

Plate A-1

TABLE A-12

2040 HURRICANE SURGE HEIGHTS

Location	Stages in feet NGVD SPH	100-year	10-year
Bayou Barataria	9.6	7.7	4.2

Levee heights for future conditions were determined by adding runup from the appropriate wave condition to the design stillwater level. Where protective structures will be sheltered against significant wave runup, wave runup from the small locally generated wave climate was used to determine levee height. On the eastern side of the study area wave berms will have to be added to maintain the same level of protection as the original project due to the loss of the woods and marsh on the flood side of the levee. In these areas where significant hurricane wave action will occur because of an available fetch, levee heights were designed using wave height determined from methodologies described in the Coastal Engineering Center's Shore Protection Manual. Design elevations of protective structures in each reach are given in Table A-13.

TABLE A-13

2040 DESIGN ELEVATION OF PROTECTIVE STRUCTURES

Location	SWL ft	WAVE RUNUP	10-year
Bayou Barataria	4.25	2.0	6.5
Eastside Levee (w/berm)	4.25	2.5	7.0

* Ground surface elevation is 0.2 ft lower.

A.10. RISK ANALYSIS.

a. Introduction. The Fisher Basin, Jean Lafitte, La., Feasibility Study's risk analysis procedures were the same as the Harvey Canal to Westwego Hurricane Protection Project Post-

Authorization Change Study (Lake Cataouatche). The Harvey Canal to Westwego Hurricane Protection Project Post-Authorization Change Study (Lake Cataouatche) was the first coastal study to undergo risk-based analysis as outlined in EC 1105-2-205. The lack of guidance on risk-based analysis for projects in the coastal zone was a main concern at that time. In a meeting held in early February 1994, officials from HEC, IWR, OCE, LMVD, and NED decided that the study was similar to a flood control study and should generally follow a riverine risk-analysis approach. It was determined that the primary effort of H&H Branch was to establish the confidence limits for the exterior stage-frequency curve. Representatives of HEC and IWR stressed that the analyses remain simple. Thus, the stage-damage function for an interior ponding area is fixed relative to the exterior stage and its confidence limits for that particular frequency. A program for non-analytical frequency curves developed by HEC extrapolated the stage-frequency curve to the far extremities and computed the standard error of the curve based on the equivalent record of the primary gage used in the basin. The output from this program was supplied to Economics Branch to use in their analysis.

b. General. Stage frequency curves cannot be described by an analytic distribution. Analysis of these curves is usually performed graphically or non-analytically. The uncertainty in a non-analytical frequency curve that is estimated from a graphical fit of ordered observations (e.g. peak annual stages) may be calculated from order statistics. No assumption has to be made concerning the analytic form of the frequency curve. The statistic derived to estimate uncertainty is termed "non-parametric" or "distribution free".

The order statistic approach is limited to calculating uncertainty in the estimated frequency curve for the range of observed data or, alternatively, the equivalent length of record. Extrapolating the estimates beyond the range of data is performed by using asymptotic approximations of uncertainty distributions. The order statistic and asymptotic estimates of uncertainty are matched at the limits of the observed data. The estimates of uncertainty are computed using the asymptotic approximation beyond the range of data.

c. Computer Program. The FORTRAN program "LIMIT", developed by HEC, was used in the computation of confidence limits. The program can be used when a frequency curve has been developed based on 1) systematic observations, 2) hypothetical events or 3) both. Input data consists of systematic observations, equivalent years of record, and the systematic and equivalent record. Output consists of 1) computation results, 2) an ASCII data file containing results that are used by the @RISK program, and 3) an HEC-DSS file that can be used to plot the frequency curve and computed confidence limits.

d. Application. The Bayou Barataria at Lafitte gage was used most extensively for this study. The lower end of the stage-frequency curve reflects the historical record and the upper end of the stage frequency curve is based on WIFM results that were calibrated to the Lafitte gage.

The equivalent record length was determined by using the guidelines as set forth in ETL 1110-2-205, dated November 1993, with the analysis setting being a long-period gage within the watershed and the model calibrated to the gage-based curve. This suggests the use of 50% to 90% of the record length. The 100-year, 200-year, and 500-year stages are hypothetical stages developed from WIFM runs.

e. Results. Confidence is high in the lower end of the stage-frequency curve. The computed error is very small between the 99.9% chance exceedence and the 50% chance exceedence. At the 50% chance exceedence and continuing to the .01% chance exceedence the confidence limits start to diverge significantly. The computed error increases from 0.063 feet at the 50% chance exceedence to over 3 feet at the .01% chance exceedence. This is expected because the less frequent events are based on hypothetical results and not experienced events.

A.11. INTERIOR DRAINAGE.

The Fisher School Basin is a subbasin of the Barataria Basin and is located on the west bank of the Mississippi River in Jefferson Parish. It is an elongated area along Bayou Barataria bounded by a local levee along the east and south, and Bayou Barataria to the west and north. High ground along Bayou Barataria directs runoff eastward. The Fisher School Basin study area encompasses approximately 45 acres. The low-lying areas in this region are prone to flooding from frequent rainfall events.

The HECIFH Interior Flood Hydrology Package (1991) computer program developed by the Corp's Hydraulic Engineering Center was used in the interior drainage analysis. The IFH program was designed to simplify the analysis of areas protected by levees and/or floodwalls. Rainfall, topography, pumping, exterior stages, and inflow from wave overtopping are all inputs into the program. The output consists of a stage-frequency curve in a tabular format. For the analysis of the interior drainage the study area acted as one ponding area. Once the outside stage reached the top of the protection, the interior stage was assumed equal to the exterior stage.

The general steps of the IFH program for hypothetical events started with entering the hypothetical rainfall storm depth-duration-frequency data (from TP-40) for multiple hypothetical events. Then the rainfall excess values for the interior basin were computed using the Initial-Uniform method. Next the rainfall excess was transformed into runoff hydrographs for the interior basin. The unit hydrographs were computed by the Clark unit hydrograph method. The flow into the interior basin from wave overtopping was then added for each event analyzed. The interior inflow was routed through the interior ponding area and discharged through the line of protection by way of pumping stations. Existing drainage canals will convey rainfall runoff to a new collector canal that will connects North Canal and Canal E1. The new canal will run parallel to the proposed diagonal levee alignment along the eastern project limit. Pump station efficiencies varied with the exterior stage. The program then determined the interior stage-frequency curve for each of the hypothetical events.

LITERATURE CITED

(1) Beach Erosion Board, "Shore Protection Planning and Design," Technical Report No. 4, June 1954.

(2) Saville, Thorndike, Jr., "Wind Setup and Waves in Shallow Water," Beach Erosion Board, Technical Memorandum No. 27, June 1952.

(3) U.S. Army Engineer District, Jacksonville, "Design Memorandum, Wind Tide Produced by Hurricanes: Partial Definite Project Report, Central and Southern Florida Project, for Flood Control and Other Purposes. Part IV, Supplement 2, Section 3, July 26, 1956.

(4) U.S. Weather Bureau, "Hurricane Rainfall Estimates Applicable to Middle Gulf Standard Project Hurricanes, Tracks A, C, F, D, and B, New Orleans Study, Zone B," Memorandum HUR 3-5, November 30, 1959.

(5) U.S. Weather Bureau, "Estimates of Moderate Hurricane Rainfall Applicable to Middle Gulf Standard Project Hurricanes," Memorandum HUR 3-5A, December 11, 1959.

(6) Bretschneider, C. L. and J. I. Collins: "Prediction of Hurricane Surge, An Investigation for Corpus Christi, Texas, and Vicinity," NESCO Technical Report SN-120 to U.S. Army Engineer District, Galveston, National Engineering Science Co., Washington, D.C., 1963.

(7) Environmental Science Services Administration, U.S. Weather Bureau, "Standard Project Hurricane Wind Field Patterns (revised) to Replace Existing Patterns in NHRP Report No. 33 for Zones B and C," Memorandum HUR 7-84, August 17, 1965.

(8) Bretschneider, C. L., "Prediction of Wind Waves and Setup in Shallow Water, with Special Application to Lake Okeechobee, Florida," Unpublished Paper, Texas A&M College, August 1954.

(9) National Weather Service, "Meteorological Criteria for Standard Project Hurricane and Probable Maximum Hurricane Windfields, Gulf and East Coasts of the United States, NOAA Technical Report NWS 23, September 1979.

(10) U.S. Weather Bureau, "Hurricane Frequency and Correlation of Hurricane Characteristics for the Gulf of Mexico Area, P.L. 71," Memorandum HUR Z-4, August 30, 1957.

RELOCATIONS

B.1. SUMMARY.

a. Scope. Relocation data was developed using the "1990 Louisiana Parish Pipeline and Industrial Atlas", various oil and gas maps, United States Geological Surveys (USGS) quadrangle maps, aerial photographs, and site visits. Preliminary relocation plans were developed in-house based on current project requirements. Pending approval of the Detailed Project Report, the owner of each facility will be allowed to review and comment on the preliminary relocation plans and cost estimates during preparation of detailed plans and specifications.

b. Estimated Relocation Cost. The estimated total cost for relocation of highways, pipelines, power and communication lines, and pumping stations for the proposed project is approximately $693,200.00. This total includes 5% for the owners engineering and design and 10% for the owners contract administration. Twenty-five percent (25%) for contingencies is added to the total for all relocation items except the highway ramps and detours. Contingencies for the ramps and detours are 30% and 35% respectively. Future Government expenditures in the areas of engineering, design, and contract administration have not been included in these estimates.

c. Authority for Accomplishing Relocations. Lands, easements, rights-of-way, relocations and disposal areas (LERRD's) are the responsibility of the local sponsor. The cost of acquiring the required LERRD's is included in the total project cost and is creditable toward the sponsor's share of the implementation costs. The local cost sharing responsibilities for project implementation vary based on the extent of the LERRD's. The minimum non-Federal contribution is 25 percent of the total project cost and the maximum is 50 percent. A minimum cash contribution of 5 percent of the overall project cost is also required.

B.2. FACILITIES UNAFFECTED BY THE PROJECT.

Several facilities parallel Louisiana State Highway 45 (LA 45) in the proposed levee alignment at the northern and southern edges of the project. Among those unaffected facilities are aerial power and telephone lines, and television cables belonging to Entergy, Bell South Telephone Company, and Cox Cable Company respectively. These lines appear to have enough clearance from the ground surface to accommodate the proposed levee. In addition, a Jefferson Parish 8-inch gravity (assumed) sewer line parallels LA 45. Rerouting a gravity line over the levee would render the line ineffective. Leaving the line in the levee section does not jeopardize the protection. Therefore, it is cost effective not to relocate the sewer line.

Sheetpile is proposed in the vicinity of Louisiana Highway 302 Bridge (LA Hwy 302) in order to avoid disturbing congested facilities. The facilities include a generator building and power pole at approximate station 154+35 belonging to Louisiana Department of Transportation and Development (LADOTD) and a power pole with power lines belonging to Entergy. Between approximate station 154+50 to 157+00 is an asphalt recreational walking track belonging to the Town of Jean Lafitte. At approximate station 155+85 along the edge of bank is an Entergy power pole where a power cable goes underground and crosses Bayou Barataria.

Four power poles, located at approximate stations 147+95, 148+40, 148+70 and 167+58, are not impacted by the current levee alignment.

There are six existing pumping stations within the proposed levee alignment that are maintained by Jefferson Parish. Three of the six pump stations discharge pipes have invert elevation above the required flowline elevation of 5.0 ft. NGVD at the point where they cross the proposed levee crown. Since these discharge pipes meet this requirement, they are unaffected. The unaffected pumps are located at approximate stations 0+00 by Fleming Canal, 92+32 by end of Oak Drive, and 159+65 by south end of Church Street.

B.3. DESCRIPTION OF IMPACTED FACILITIES, PROPOSED RELOCATIONS, AND COSTS. The estimated relocation costs given in the following description do not include contingencies, owners engineering and design, and owners contract administration. Refer to section B.1.b. for those cost estimates.

a. Highways. The estimated total relocation cost for highways are $372,120.00. The following highways cross the proposed levee alignment, and require relocation:

Louisiana State Highway 45 (LA 45). Louisiana State Highway 45 (LA 45) traverses the proposed levee alignment at the northern and southern edges of the project, and will require relocation. This is a two-lane, asphaltic concrete, through traffic primary highway. Ramps will be constructed to raise the two reaches to the project flood protection level of +7.0 feet NGVD. The approximate length of the proposed ramps will be approximately 1200 feet each.

Temporary detours will be constructed to allow continuation of traffic during construction. Due to the limited area available for a detour road at this location, we anticipate a phased construction of the ramp with a single lane detour will be necessary. Flagmen will be required during construction to direct traffic. Each day, following construction, the contractor will be required to restore the work area to a driveable condition that will allow two-lane highway traffic. The cost estimates for the two new LA 45 ramps is $236,290.00, and the two detours is $135,830.00.

b. Pipelines. The estimated total cost for the relocation of affected pipelines is $47,120.00. The following pipelines cross the proposed levee alignment, and require relocation:

(1) Jefferson Parish. Two 8-inch waterlines run parallel to LA 45, with one line on each side. These lines cross the proposed alignment of the levee at the northern and southern edges of project. The estimated relocation cost is $28,720.00. This estimate is based on rerouting lines over the new levee.

(2) Louisiana Gas Service Company. The Louisiana Gas Service Company owns a 3-inch gas pipeline that runs parallel to LA 45. This line crosses the proposed levee alignment at the northern and southern edges of project. The estimated

relocation cost is $11,400.00. This estimate is based on rerouting the line over the new levee.

(3) U.S. Oil and Gas Incorporated. U.S. Oil and Gas Inc. owns a 2 1/2-inch abandoned pipeline that runs parallel to the existing levee on the north-eastern side of project at the end of Oak Drive. The estimated cost for removing the section of pipeline located within the proposed levee alignment is $7,000.00.

c. Power and Communication Lines. The estimated total cost for the relocation of affected power and communication lines is $38,120.00. The following powerlines, poles, and telephone cables are within the proposed levee alignment, and require relocation:

(1) Entergy Louisiana Inc. Electrical Power Service (Entergy). Entergy owns the following facilities:

(a) Powerlines and pole located north of Fleming Canal pumping station at approximate baseline station 1+40.

(b) Powerlines and pole located south of Fleming Park Road by Dufrene Street at approximate baseline station 13+34.

(c) Powerline and pole parallel to Gloria Drive at approximate baseline station 73+53.

(d) Powerline and pole south of Fleming Canal pumping station at approximate baseline station 182+88.

The estimated total cost for relocation of Entergy powerlines and poles is $15,080.00. This estimate is based on moving poles out of the proposed levee alignment, and detaching and reattaching associated electrical service lines.

(2) BellSouth Telecommunication Inc.(BellSouth).

(a) Underground BellSouth telephone cables that run parallel to LA 45 are affected at the northern and southern edge of project. The estimated relocation cost is $3,700.00. This estimate is based on rerouting cable over the new levee.

(b) One underground BellSouth telephone cable crossing Bayou Barataria at approximate station 151+75 is affected. This line crosses the bank at a proposed sheetpile floodwall location and will have to be sleeved through the sheetpile. The estimated relocation cost is $1,000.00.

The estimated total cost for relocation of BellSouth communication cables is $4,700.00.

(3) Jefferson Parish, LA. Jefferson Parish owns a powerline and pole located on the northeast side of Gloria Drive at approximate baseline station 72+33. They also own an electrical power/control station located south of, and associated with, the Fleming Canal pumping station at approximate baseline station 182+88. This station consists of a fenced-in antenna pole, power pole with electric panels, and a 4-feet by 6-feet concrete slab. The estimated cost for relocation of Jefferson Parish facilities is $18,340.00. This estimate is based on relocation of the electrical power/control station outside of the proposed levee alignment, and detaching and reattaching associated electrical service lines.

d. Drainage Pump Stations. The invert elevation of all discharge pipes running from pumping stations within the project must be above the flowline elevation of 5.0 ft. NGVD at the point where they cross the proposed levee crown. Discharge pipes from two of the five existing pumping stations fail to meet these requirements, and therefore have to be modified. The 24-inch discharge pipe associated with the Gloria Drive pumping station at approximate baseline station 71+63, and an 18-inch discharge pipe associated with the Perkins Street pumping station at approximate baseline station 175+57 will have to be raised to an invert elevation above 5.0 NGVD. The estimated relocation cost is $4,500.00.

STRUCTURAL DESIGN

C.1. INTRODUCTION.

Proposed flood protection for the Fisher School Basin, adjacent to the east bank of Bayou Barataria, will consist of a combination of an existing earthen levee along the eastern and southern study area boundaries that will be heightened and construction of new reinforced concrete floodwalls. The proposed flood protection system will encircle the basin and provide protection up to elevation 7.0 ft. NGVD.

Water elevations within the basin's protection system will be maintained by a series of existing pumping stations. Louisiana Highway 45 will be relocated at the northeastern and southern project limits by ramping over the earthen levee section along the highway's existing alignment in order to provide access to the protected area. The reinforced concrete floodwalls will be located in eight reaches interspersed by levee sections, and shall consist of I-Type, inverted T-Type and Bulkhead-Type walls. Approximately 7,316 linear feet of I-type, T-type and Bulkhead-Type floodwalls will be located in Reaches #1 through #6. Reaches #7 and #8 will contain a total of approximately 254 linear feet of I-Type floodwalls that is expected to be built integrally with two of the existing pumping stations. Reach #4 will contain two 30 feet lengths of uncapped steel sheetpiling located at two separate pumping stations for a total of 60 feet. The total length of structural flood protection for Reaches #1 through #8 will be approximately 7,630 feet.

From studies of aerial photographs of the area and site visits to the proposed flood protection alignment, several locations were identified where access from the protected side of the wall to the floodside of the wall will be provided. Needed access through the flood protection ranges from reinforced concrete stairs over the floodwall to vehicular and pedestrian swing type floodgates. The T-type floodwalls shall contain vehicular access gate openings that are capable of being closed during flood stages by means of hinged steel swing-type floodgates. (See Plate 15). The proposed floodwall alignment for this project will require a total of 11 access gates and 25 sets of access stairs. The access gates shall consist of one 5-feet opening pedestrian gate, eight 15-feet opening vehicular gates and two 30-feet opening vehicular gates.

The use of a Bulkhead-Type of wall was determined to be best suited for use in areas adjacent to existing bulkheads. Many of the existing bulkheads are anchored into their locations by means of buried anchor guy wires. There is no cost effective method for determining the number of bulkheads constructed using this method. Construction of a conventional I-Type floodwall would result in driving steel sheetpiling through the anchor guy wires, which would result in failure to the existing bulkheads unless they were somehow shored prior to construction. Given the expense of this type of shoring and the unknown number of bulkheads affected, the Bulkhead type of floodwall was considered to be the best option. In areas where the Bulkhead-Type of floodwall is proposed, no land acquisition is anticipated.

C.2. FLOODWALL DESIGN

a. I-Type Floodwall. The landside I-wall consists of a reinforced concrete cap encapsulating the top 3 feet of continuously interlocked steel sheetpiling. The steel sheetpiling shall have an approximate embedment to stick-up ratio of 3:1 and will provide stability for the I-wall as well as cut-off protection against under seepage. The concrete cap will have a uniform thickness of 21" and will generally extend from 2 feet below the existing natural ground up to elevation 7.0. Modified I-walls will contain small openings in the concrete above the adjacent ground surface that will be capable of being closed by means of steel swing gates mounted on the flood side of the walls. (See Plate 17)

b. Bulkhead-Type Floodwall. The Bulkhead walls will be constructed adjacent to the bayou side of existing bulkheads and shall consist of a 21 -inch wide reinforced concrete cap encapsulating the top portion of continuously interlocked steel sheetpiling. The steel sheetpiling for the bulkhead walls shall be designed to act as a cantilever retaining walls and shall have a clear distance of 2 feet to the existing bulkheads. The void between the existing bulkheads and the Bulkhead-Type floodwall will be backfilled with earthen material, and the concrete cap on the flood side of the bulkhead wall will extend 2 feet downward from the top of floodwall elevation. On the protected side of the bulkhead wall, the concrete cap will extend down from the top of the wall to a distance of 6 inches below the adjacent natural ground.

c. T-Type Floodwall. T-wall gate monoliths consist of reinforced concrete "Inverted T-type" monoliths, 25 feet in length for the 15-feet opening gates and 40 feet in length for the 30-feet opening gates, with bases supporting 21" thick reinforced concrete walls. The bases of each monolith are 8 feet wide by 2 1/2 feet thick and are supported by ten, and sixteen 12-inch diameter timber piles for the 15-feet and 30-feet opening gate monoliths, respectively. T-walls will be used in lieu of I-walls at locations where vehicular access through the flood protection system is required. Continuously interlocked steel sheetpiling that will tie into the adjacent I-walls will be located longitudinally along the bottom centerline of the base for cut-off protection against under seepage. Column sections two feet wide located adjacent to each side of the opening shall support the hinged steel swing gates in the open and closed positions. (See Plate 12)

C.3. OTHER PROJECT FEATURES

a. Swing-Type Floodgates. Hinged steel swing-type gates at vehicular and pedestrian access openings in the floodwall shall be located as shown on the drawings for the purpose of closing the openings in the flood protection system during high water stages. Swing gates shall be mounted by a hinge and pedestal to the floodside of the column sections and shall be stored back against the adjacent I-wall sections when in the opened position. (See Plate 13) A typical steel swing gate is a steel frame that consists of horizontal wide-flange main members at the top and bottom of the gate that are connected by vertical ribs and stiffener plates and is covered with a 5/16" thick skin plate (See Plate 15). Seal details built onto the steel swing gates shall be capable of providing watertight seals with the seal plates cast into the T-wall monolith along the sides and bottoms of the gates when in the closed position. Provisions will be made for locking the swing gates in both, the open and closed positions.

b. Access Ladders and Stairs. Steel ladders shall be provided at each gate location in order to allow personnel closing the steel swing gates during high water stages to have access back to the protected side of the floodwall after the gates have been closed. Ladders shall be hot-dipped galvanized after fabrication (See Plate 14). Reinforced concrete stairs will be constructed integrally with the landside I-Type floodwall at locations where a resident’s ready access to existing facilities adjacent to the waters edge has been interrupted.

FOUNDATION INVESTIGATION & DESIGN

D.1. GENERAL

This section describes the soil investigation and design for approximately 32,000 linear feet of improved levee located in the Fisher School Basin, Jean Lafitte, LA.

D.2. FIELD INVESTIGATION AND LABORATORY TESTING

Four (4) hand auger borings, 20 feet deep, were taken along the proposed levee and floodwall alignment. A visual classification of all samples obtained from the borings was conducted and the soil properties and stratification were then estimated from these classifications. During the next phase of the project, approximately 17 undisturbed borings will be obtained along the proposed flood protection alignment. At each floodgate location, one boring at least 60 feet deep will be acquired (for a total of 10 borings). The remaining seven (7) borings will be acquired along the levee alignment every 2500 feet at minimum depths of 30 feet. Visual classifications, atterberg limits, and unconfined compression, triaxial and consolidation tests will then be performed on selected samples.

D.3. FOUNDATION CONDITIONS

The design stratification as determined from the above mentioned borings consists primarily of soft clays with lenses of silt and a layer of silt from approximate elevation –10.0 ft. to –15.0 ft. NGVD. Definitive design stratification will be established using the undisturbed borings proposed for the next phase of this project.

D.4. STABILITY ANALYSES

a. Levees. Using survey cross sections and hand auger boring data, stability analyses were performed for composite design sections of the proposed levee embankment. This analysis was accomplished using the Lower Mississippi Valley Division (LMVD) Method of Planes Stability Analysis Program. The levee is designed for a minimum factor of safety of 1.30 which resulted in a required cross section consisting of a 5.0 foot wide levee crown with 1 on 4 side slopes. During preparation of detailed plans and specifications, data acquired from undisturbed borings will be used to verify the stability analysis. A soils report containing the data acquired during the subsurface investigation will be prepared.

b. Cantilever I-Wall (Floodwall). I-wall stability and required penetration were determined by the "Method of Planes". A "Factor of Safety" was applied to the soil parameters. For the friction angle, the F.S. was applied as follows:

$$F_d = \tan^{-1}\left(\frac{\tan F_a}{\text{factor of safety}}\right)$$

where, F_a = available friction angle

F_d = developed friction angle

The developed friction angle was used in determining lateral earth pressure coefficients. Using the resulting shear strengths, net horizontal water and earth pressure diagrams were determined for movement toward each side of the sheet pile. From the earth pressure diagrams, a summation of horizontal forces were equated to zero and a summation of overturning moments were determined for various tip penetrations. The depth of necessary penetration is the point of zero summation of moments. The following design cases were analyzed for determining required penetration for the levee/I-walls.

No significant wave load on I-wall:

<u>Q-Case</u>

F.S. = 1.5 with static water at still water level (SWL)
F.S. = 1.0 with static water at (SWL) plus 2 feet
General: If the penetration to head ratio is less than 3:1, then increase it to 3:1.

The cantilevered I-wall analysis will be rerun to verify the floodwall tip penetration using undisturbed boring test results and amended soil stratification in the next project phase.

D.5 PILE CAPACITY CURVES.

The pile capacity curves for concrete and timber piles (ranging from 12-inch to 18-inch) used to support the proposed floodgates were derived to illustrate the ultimate pile capacities at various depths. These pile capacities will be verified during the development of detailed plans and specifications using the data acquired from undisturbed borings or pile test data as appropriate.

D.6. SETTLEMENT.

The consolidation of levee embankment and floodgates will be analyzed in the next project phase. Consolidation tests will be performed on soil samples acquired from undisturbed borings. For each consolidation test, the compression index, C_C vs. elevation will be plotted to show the range of values at various depths. The settlement due to the proposed levee and floodwall will then be determined and the required embankment and floodwall overbuild estimated.

D.7. SEEPAGE CONTROL.

A sheetpile cutoff will be installed beneath each floodgate to an elevation, which will be determined via seepage analysis during the next phase of this project.

COST ESTIMATES

E.1. INTRODUCTION

A detailed description of project cost estimates is provided in this section. The concrete capped sheetpile floodwall and earthen levee embankment cost estimates are described on page A-53. Along the western project limit, adjacent to Bayou Barataria, the flood protection project consists mainly of concrete-capped sheetpile floodwall with floodgates interspersed for water access. At the southern and eastern project limits, an existing earthen levee will be heightened and lengthened to protect the Fisher School Basin.

E.2. ENGINEERING AND DESIGN ESTIMATES

Engineering and Design (E&D) for this project consists of preparing detailed design plates for construction. Pending approval of this DPR, additional funding will be provided to develop plans and specifications. E&D cost estimates are as follows:

Geotechnical Br.	$ 56,000.00
Structures Br.	$ 81,250.00
General Engineering Br.	$ 3,350.00
Cost Engineering Br.	$ 18,000.00
Hydraulics Br.	$ 2,500.00

Civil Br.	$ 80,000.00
Design Services Br.	$ 16,000.00
Surveys	$ 90,000.00
Engr Div Total	$347,100.00
Construction Div.	$ 25,000.00
Project Mgmt. Div.	$ 40,000.00
E&D TOTAL	**$412,100.00**

E.3. SUPERVISION AND ADMINISTRATION ESTIMATES

Supervision and Administration (S&A) of the construction contracts for this project is the responsibility of the U.S. Army Corps of Engineers. S&A cost estimates are as follows:

Construction Div.	$720,000.00
Project Mgmt. Div.	$30,000.00
S&A TOTAL	**$750,000.00**